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P. S. Skjærbæk, S. R. K. Nielsen, P. H. Kirkegaard & A. Ş. Çakmak
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Case Study of Local Damage Indicators for a 2-Bay, 6-Storey RC-Frame subject to Earthquakes

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Abstract: *A simulation study of a 2-bay, 6-storey model test RC-frame (scale 1:5) subject to earthquakes is considered in this paper. Based on measured (simulated) storey accelerations and ground surface accelerations several indices for the storey damage, including interstorey drift, flexural damage ratios, normalized cumulative deformations, normalized cumulative dissipated energy, Park and Ang's indicator, a low-cycle fatigue damage index and a recently proposed local softening damage index estimated from time-varying eigenfrequencies are used to evaluate the damage state of the structure after the earthquake. Storey displacements are obtained by double time-integration of the measured accelerations, and the storey shear forces are calculated by using a simple shear model, where the shear force at a given storey is determined as the summation of inertial forces in the storeys above. All the mentioned methods except for the latter require measurements at all storeys, whereas the time-varying frequencies can be estimated from a single measurement by using system identification in combination with a moving window technique. The estimated values of the various damage indicators are then compared with damage indicators calculated by the simulation program SARCOF from the time-varying stiffness matrix of the FE-model. Good correlation was found between the SARCOF damage indicator and the considered low cycle fatigue damage indicator, the flexural damage ratio, The Park and Ang indicator and the local softening damage indicator. In the considered case, the damage state of the two lower storeys can be characterized as very severe and in general this is displayed by the various damage indicators except for the normalized cumulative plastic deformation which shows far too low a damage level in the second storey.*

Keywords: Damage Indices, Localization, Earthquakes, RC-Frames.

Nomenclature

u	Interstorey displacement.
H	Storey heigth in RC-frame.
ω	Circular eigenfrequency.
K	Stiffness
$\mathbf{K}$	Stiffness matrix.
$\mathbf{M}$	Mass matrix.
$\mathbf{\Phi}$	Mode shape matrix.
P	Force.
E	Energy.
α	Fatigue exponent.
b	Constant.
β	Constant.

1 Introduction

Experiences from past earthquakes in the last decade have shown a growing need for methods to localize and quantify damage sustained by RC-structures during earthquakes. Traditional visual inspection and field testing can be used to locate and measure the damage state of an RC-structure. However, a much more attractive method is to measure the response of the structure at one or more positions and from this information estimate the damage state of the structure. During the last 10-20 years much research has been performed within this area and many different methods for damage assessment have been suggested in the literature. Almost all of the proposed methods are based on calculating a socalled damage index, which is supposed to reflect the damage state of the considered structure, substructure or structural member. Unfortunately many of the suggested damage indices do not have a well defined mapping of the numerical value to a certain damage state, and the mapping of some of the indices has shown a significantly dependence on the considered structure which makes the index difficult to use for damage assessment. The requirements for a good damage assessment method can therefore be formulated as follows, se e.g. Stephens [13]

1. The index should have general applicability, i.e. it should be valid for a variety of structural systems.
2. It should be based on a simple formulation and be easy to use.
3. It should generate easy interpretable results.

The purpose of this paper is to investigate how the selected method for damage assessment of RC-structures suggested in the literature meets the abovementioned requirements. The investigations are performed on simulated acceleration response measurements from shaking table tests with a 2-bay, 6-storey model test RC-frame. The geometry of the model test frame is shown in figure 1. The FE-model considered is a model of a test frame to be tested at the structural laboratory at Aalborg University during the autumn 1996. In the FE-model the beam and column cross-sections are assumed to be 50mm in width and 60mm in height. The beams are assumed to be reinforced with 4 6mm KS410 ribbed reinforcement steel bars and the columns with 6 6mm KS410 ribbed reinforcement bars. The concrete strength is assumed to be 25 MPa. From preliminary analysis the two lowest eigenfrequencies of the model were found to be $f_1 = 1.93$Hz and $f_2 = 6.13$Hz.

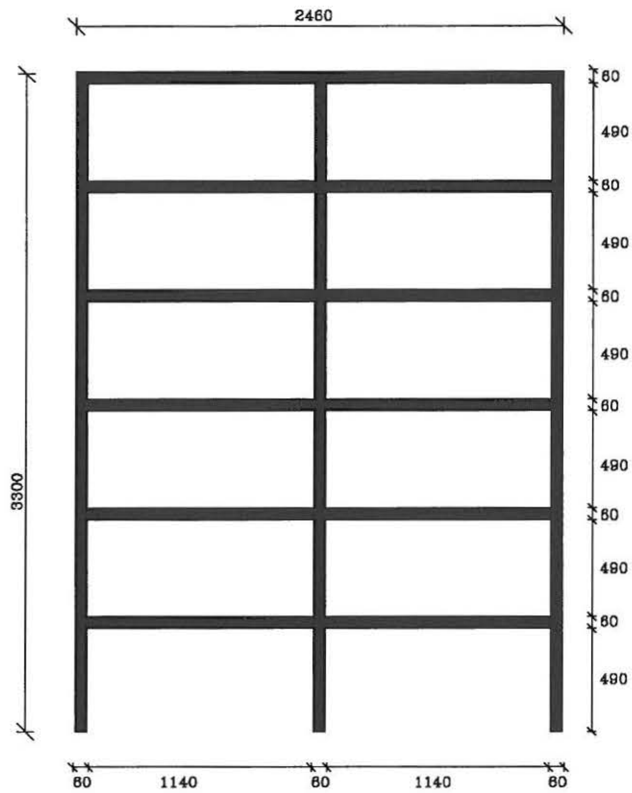

Figure 1: Geometry of the considered 2-bay, 6-storey model test frame. All measures in mm.

In this simulation study of the shaking table tests it is asssumed that the model test frame was instrumented with accelerometers at all storeys and at the base providing time series of storey accelerations and base acceleration. Since the methods investigated in this paper require different input, the measured signals have been processed in various ways. Generally the examined damage indices can be identified from one or more of the following quantities:

- Displacements
- Restoring forces
- Eigenfrequencies

Displacements are obtained by filtering, detrending and double integration of the acceleration signal, shear forces are obtained by using a simple shear spring model, where the shear force are calculated as the summation of inertial forces in the storeys above the considered storey. Frequencies are estimated by fitting an ARMA model to one of the acceleration responses using a windowing technique to capture the time-varying effects, see Kirkegaard et al. [5].

2 Considered Damage Indices

It is the aim to select the damage indices considered so that all basic measures of damage are represented. In the large variety of methods proposed in the literature these have been formulated in various ways, and the ones used here have been selected quite arbitrarily. A more throroughly overview of the damage indices suggested in the literature can be found in Stephens [13] or more recently in Williams et al. [15]. I should be noted, that the formulation presented in the following presumes that the methods are devised for assessment of storey damage in a frame structure.

2.1 Interstorey Drift, ID

Damage indices based on interstorey drifts have been proposed in various formulations by Culver et al. [3], Toussi and Yao [14], Sozen [12] and Roufaiel and Mayer [8]. The index based on interstorey drift considered here is due to Toussi and Yao, who defined their index for the ith storey as the ratio between the maximum interstorey drift $u_{\max,i}$ and the storey height H as

$$ID_i = \frac{u_{\max,i}}{H} \tag{1}$$

From studies of test data of structural components and small-scale structures, it was found that the values of ID_i smaller than 1% correspond to damage of non-structural components while values larger than 4% may result in irrepairable damage or collapse.

2.2 Normalized Cumulative Deformations, NCD, and Dissipated Energy, NDE

Banon et al. [1] defined a damage index based on cumulative plastic deformation as the ratio of the sum over all n half-cycles of all maximum plastic deformations to the deformation at yield $u_{y,i}$ at the ith storey

$$NCD_i = \sum_{j=1}^{n} \frac{|u_{p,j}|_i}{u_{y,i}} \tag{2}$$

Normally the maximum plastic deformation in a half-cycle is calculated as the displacement at zero force in the force-deformation curve. Generally no rule has been developed for mapping values of this index to an actual damage state of the structure.

Along with the normalized cumulative deformations Banon et al. [1] also considered the normalized cumulative dissipated energy as a damage index, which was defined as the ratio of the energy dissipated in inelastic deformation to the maximum elastic energy that would be stored in the member in anti-symmetric bending

$$NDE_i = \int_0^t \frac{2P_i(\tau)u_i(d\tau)}{P_{y,i}u_{y,i}} \tag{3}$$

where $P(\tau)$ is the shear force at the time τ, $u_i(d\tau)$ is the deformation from $t = \tau$ to $t = \tau + d\tau$, $P_{y,i}$ is the yield force for the ith storey and $u_{y,i}$ is the yield deformation. As for the cumulative deformations, a rule for mapping given values of the index into a specific damage state is lacking.

2.3 Flexural Damage Ratio, FDR

In another suggestion, Banon et al. [1] correlated damage to the ratio of initial stiffness $K_{i,i}$ to the reduced secant stiffness $K_{RS,i}$ at the maximum displacement given as

$$FDR_i = \frac{K_{i,i}}{K_{RS,i}} \tag{4}$$

An example of how the Flexural Damage Ratio is calculated is illustrated in figure 2.

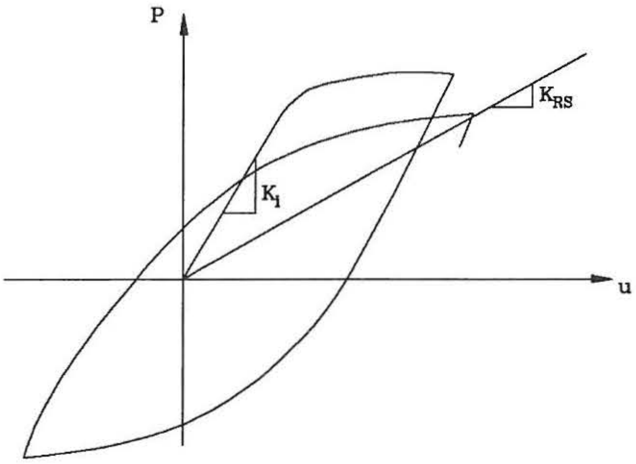

Figure 2: Definition of flexural damage ratio.

2.4 Stephens' Extended Index, SEI

Stephens [13] defined a cumulative plastic deformation damage index where the damage sustained during the jth half-cycle of response is determined as

$$\Delta d_j = \left(\frac{\Delta\delta_{pt}}{\Delta\delta_{pf}}\right)^{\alpha} \tag{5}$$

where $\Delta\delta_{pt}$ is the positive change in plastic deformation, $\Delta\delta_{pf}$ is the positive change in plastic deformation in a one-cycle test to failure conducted at the relative deformation ratio, rl, of cycle j. The relative deformation ratio is defined as the ratio of the negative change in plastic deformation in cycle j, $\Delta\delta_{pc}$, to the positive change in plastic deformation in plastic deformation in cycle j. α is a fatigue damage exponent given as $\alpha = 1 - (b \cdot rl)$. Stephens suggested the value $b = 0.77$ to be used for RC-components. The parameters in Stephens index are defined in figure 3.

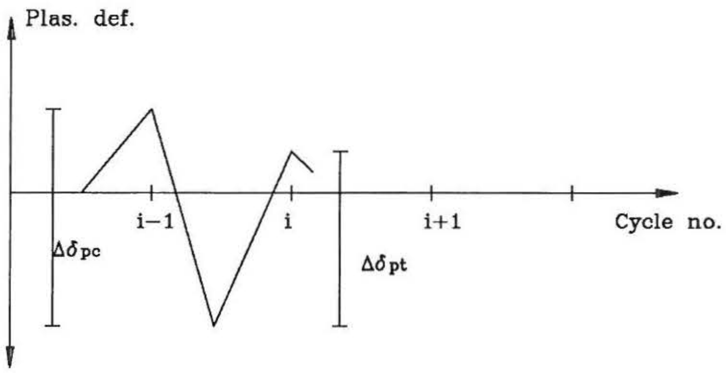

Figure 3: Definition of parameters in Stephens' index.

The total damage of the ith storey is then obtained by linear summation of the damage contribution of all half-cycles.

$$SEI = \sum_{j=1}^{n} \Delta d_j \tag{6}$$

2.5 Park and Ang's Index, P&A

Park and Ang's [6] index combines the contributions, from maximum deformation damage and from dissipated energy as

$$P\&A_i = \frac{u_{\max,i}}{u_{u,i}} + \frac{\beta}{P_{y,i} u_{u,i}} \int dE_i \tag{7}$$

where $u_{\max,i}$ is the maximum deformation in storey i during the earthquake, $u_{u,i}$ is the ultimative deformation under monotonic loading, $P_{y,i}$ is the yield strength, dE_i is the incremental dissipated energy and β is a non-negative strength deterioration parameter, which on average has been found to be 0.25. On average it is supposed that a value of 1 of this index corresponds to collapse.

2.6 Local Softening Damage Index, LSDI

The local softening damage index has recently been suggested by Skjærbæk et al. [9], and is based on the socalled maximum softening damage index originally proposed by DiPasquale et al. [4] which is a global damage index calculated from the maximum reduction in the smoothed fundamental period of the structure.

The local softening damage index $LSDI_i$ for substructure i is defined from

$$\mathbf{K}_{i,e}(t) = (1 - LSDI_i)^2 \, \mathbf{K}_{i,0} \tag{8}$$

where $\mathbf{K}_{i,0}$ is the initial undamaged stiffness matrix of the substructure and $\mathbf{K}_{i,e}(t)$ is the equivalent stiffness matrix for which the summation over all storeys

$$\mathbf{K}_e(t) = \sum_{i=1}^{n} \mathbf{K}_{i,e}(t) \tag{9}$$

produces an equivalent global stiffness matrix $\mathbf{K}_e(t)$. $LSDI_i$ is then identified so that $\mathbf{K}_e(t)$ produces exactly the measured smoothed eigenfrequencies $f_i(t)$, $i = 1, 2$ at a given time t. The LSDIs for each storey are solved from the equation

$$(\mathbf{K}_e(t) - (f_i(t))^2 \mathbf{M}) \mathbf{\Phi}_i(t) = \mathbf{0} \tag{10}$$

Since normally more than two LSDI's have to be dermined, these cannot be determined uniquely if only two eigenfrequencies are identified, and a special technique has to be used. The method used here is thoroughly described in Skjærbæk et al. [9], [10].

2.7 SARCOF Damage Indicator

For all simulations performed, the SARCOF program has been modified to calculate what could be considered to be the "real" damage of each element as a function of time. This damage index is referred to as δ_{SARCOF} and is calculated from an equivalent homogeneous bending stiffness $\langle EI(t)\rangle$ of the element as

$$\langle EI(t)\rangle = (1 - \delta_{SARCOF}(t))^2 EI_0 \tag{11}$$

giving

$$\delta_{SARCOF}(t) = 1 - \sqrt{\frac{\langle EI(t)\rangle}{EI_0}} \tag{12}$$

where EI_0 is the bending stiffness of the undamaged beam. The equivalent homogeneous bending stiffness is calculated as

$$\frac{l}{\langle EI(t)\rangle} = \int_0^l \frac{dx}{EI(x,t)} \tag{13}$$

where l is the length of the element. This damage index is used as the actual or "real" damage of the element to evaluate the damage indices obtained by the proposed method. $\langle EI(t)\rangle$ is the bending stiffness that produces the same deflection of a beam to bending moments of equal magnitude applied at the end-sections as the actual beam element. When the damage localization is performed at storey level, the damage indices in all elements in each storey are weighted into a storey damage index using the method presented in the following section.

2.8 Weighting of Element DIs

Park et al. [6] suggested that the local damage index itself is used as weight so that

$$D_g^{DI} = \frac{\sum_{i=1}^n D_i^2}{\sum_{i=1}^n D_i} \tag{14}$$

It should be noted that (14) are only an example of possible weights that can be used to calculate a global damage index from local damage indices and that there is no unique mapping from local to global damage. The weights could also be assigned from considerations such as lower storeys are more important than upper storeys, etc.

3 Results

The finite element model of the frame was exposed to the ground motions shown in figure 4.

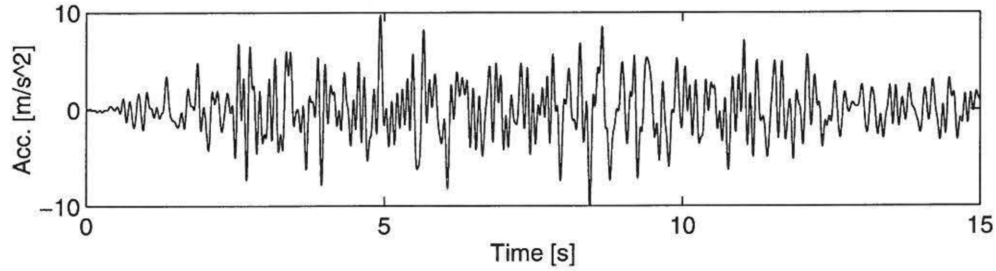

Figure 4: The ground motions, applied to the structure.

In figure 5 the corresponding total storey accelerations are shown.

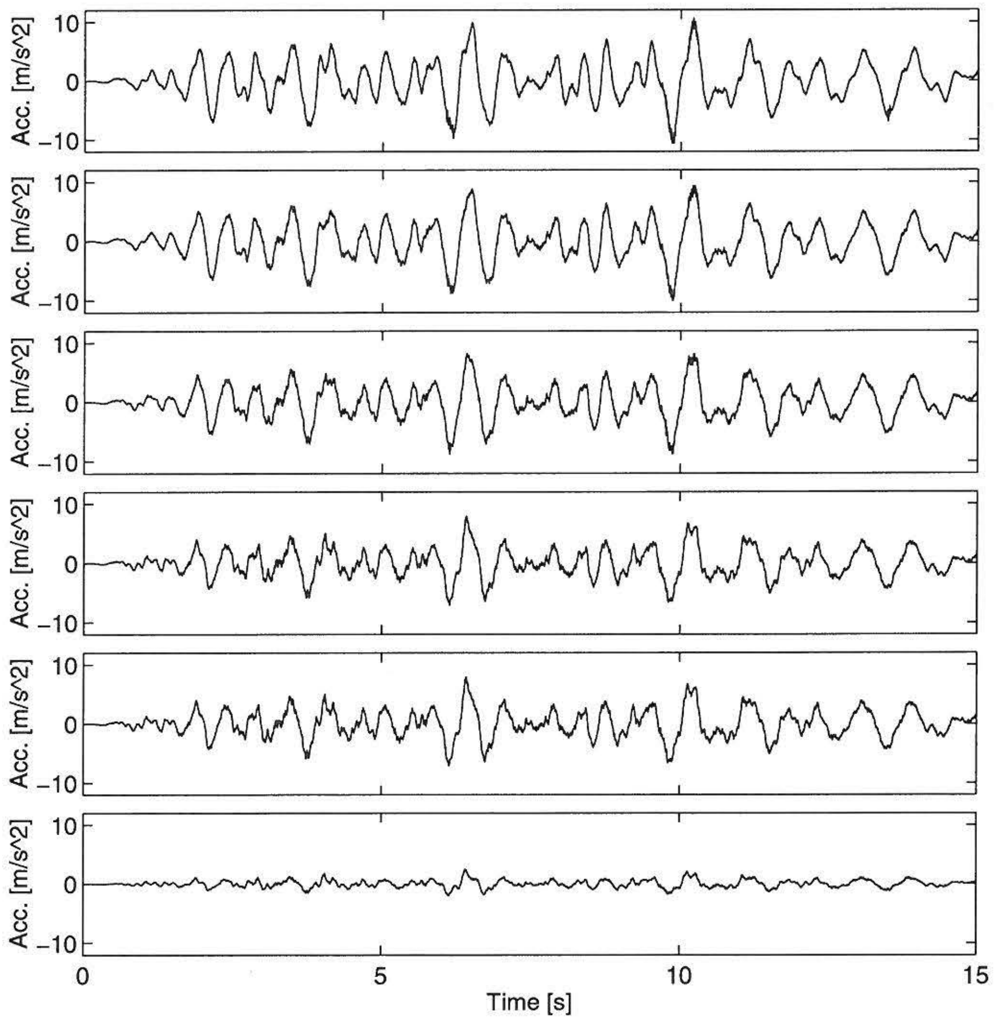

Figure 5: Measured storey accelerations.

3.1 Estimation of Force-Deformation Curve

Stephens [13] used a relatively simple method to estimate the interstorey force-displacement curve using acceleration response information. This method works in three steps. The lateral restoring force is calculated in a shear model of the structure using the acceleration data. The corresponding deformations are obtained from the displacement response which is obtained from noise treatment and integration of the acceleration data. The force-deformation response is estimated from this information using a least squares interpolation technique.
Using a multi-degree-of-freedom shear model assigned one lateral degree of freedom at each measuring point (storey), where the storey mass is lumped the shear force can be determined from storey accelerations. This is illustrated in figure 6.

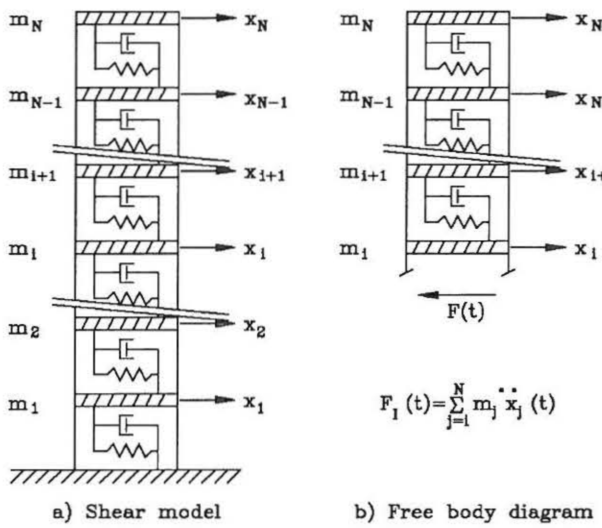

Figure 6: Simple shear model.

The dynamic equilibrium expression will then be on the form

$$F_i(t) = \sum_{j=i}^{N} m_j \ddot{x}_j(t) \tag{15}$$

for each interval between the storeys. $F_i(t)$ is the shear force in the storey below mass i, N is the total number of masses, m_j is the mass of storey j and $\ddot{x}_j(t)$ is the measured absolute acceleration at storey j. The shear forces can the be calculated inserting the measured accelerations into eq. 15.
The deformation at each storey is calculated by taking the difference of the corresponding storey displacements obtained from integration of the measured accelerations.
The force-deformation response between adjacent masses of the structure is then estimated by a polynomial piecewise paired force and deformation histories, see figure 7.
I should be noted, that this method is only effective for structures where the relative displacement directly displays the deformation behaviour as in the case of the frame structures illustrated in figure 8.
In order to evaluate the displacement from measured acceleration time-series, two time-integrations of the acceleration response become necessary. This process was intensively studied by Stephens [13] and he concluded that if the noise filled acceleration signal was used uncritically very misleading results may be obtained. Stephens [13] suggested that the acceleration signal was bandpass-

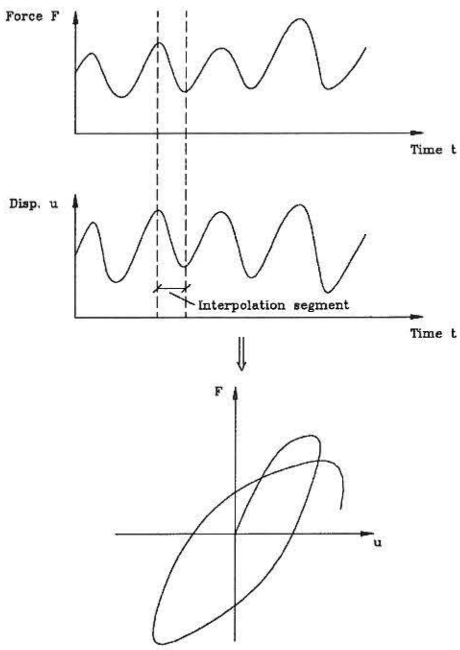

Figure 7: Piecewise paired force and deformation giving the force-deformation curve.

filtered to cut very low and high frequency components out of the signal before integration. After the first integration the velocity response was obtained and a new bandpass-filtering was performed before the last integration to obtain the displacement response. The procedure to change from acceleration response to displacement response is illustrated in figure 9.

3.2 Estimation of frequencies

The two lowest eigenfrequencies of the degrading system are identified using a scalar ARMA model on a running window of the measured ground excitation and the top storey response. Such models have been applied by numerous investigators for identification of frequencies of time invariant systems, see Skjærbæk et al. [11]. DiPasquale et al. [4] investigated the influence of the window length on the estimated value of the lowest eigenfrequency of the structure and concluded that

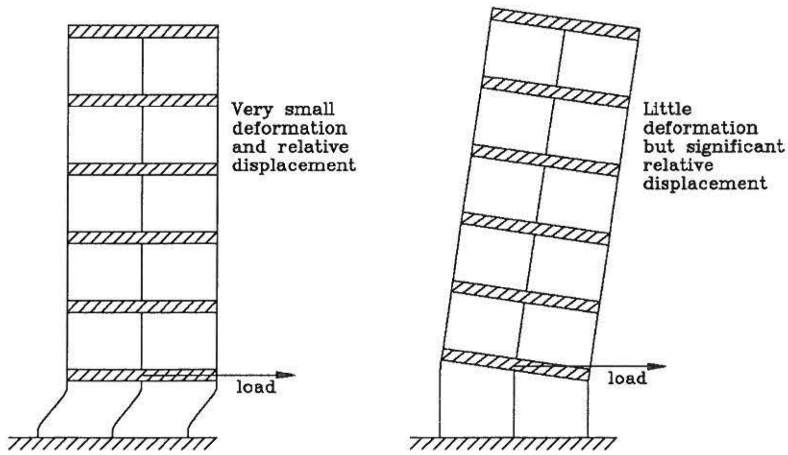

Figure 8: Relative displacement versus deformation response of frame and shear wall structure

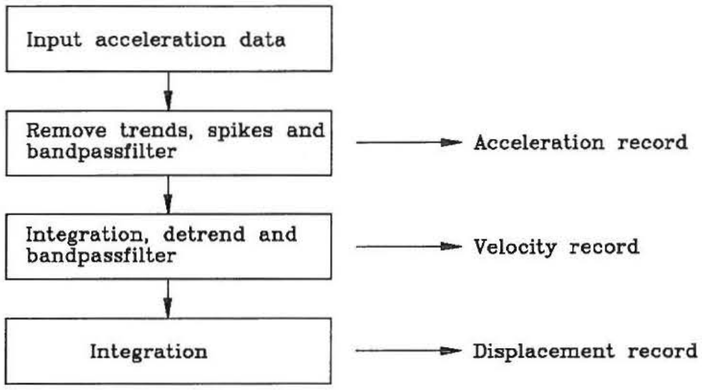

Figure 9: The procedure to transform acceleration response to displacement response.

a window length of 2.4 times the fundamental period of the undamaged structure gave resonable estimates.

In this study an ARMA model with 4 autoregressive terms and 2 moving average terms was used. At the time of maximum reduction in the lowest frequency, $f_1 = 1.14$Hz and $f_2 = 4.60$Hz were found.

3.3 Calculated DIs

By processing these series, the force-deformation curves for each storey can be obtained. These are shown in figure 10.
From figure 10 the damage indices in section 2 can be calculated, and these are listed in table 1 along with the damage indices calculated by SARCOF.

	1st	2nd	3rd	4th	5th	6th
ID	0.048	0.071	0.060	0.040	0.021	0.009
NCD	24.36	8.39	8.15	4.56	0.77	1.23
NDE	43.24	25.56	17.58	8.37	0.48	0.18
FDR	3.71	2.12	1.87	1.51	1.27	1.14
SEI	0.69	0.47	0.42	0.16	0.03	0.04
P& A	0.95	1.29	1.00	0.59	0.23	0.11
LSDI	0.50	0.49	0.38	0.00	0.00	0.00
SARCOF	0.49	0.46	0.37	0.18	0.03	0.00

Table 1: *Values of damage indices.*

Where the following parameters have been used when calculating the DIs.

A failure interstorey drift of 10 per cent
$\beta = 0.25$ in Park and Ang's index
$\beta = 0.77$ in Stephens extended index SEI

Table 2: *Used parameters for calculation of the various damage indicators.*

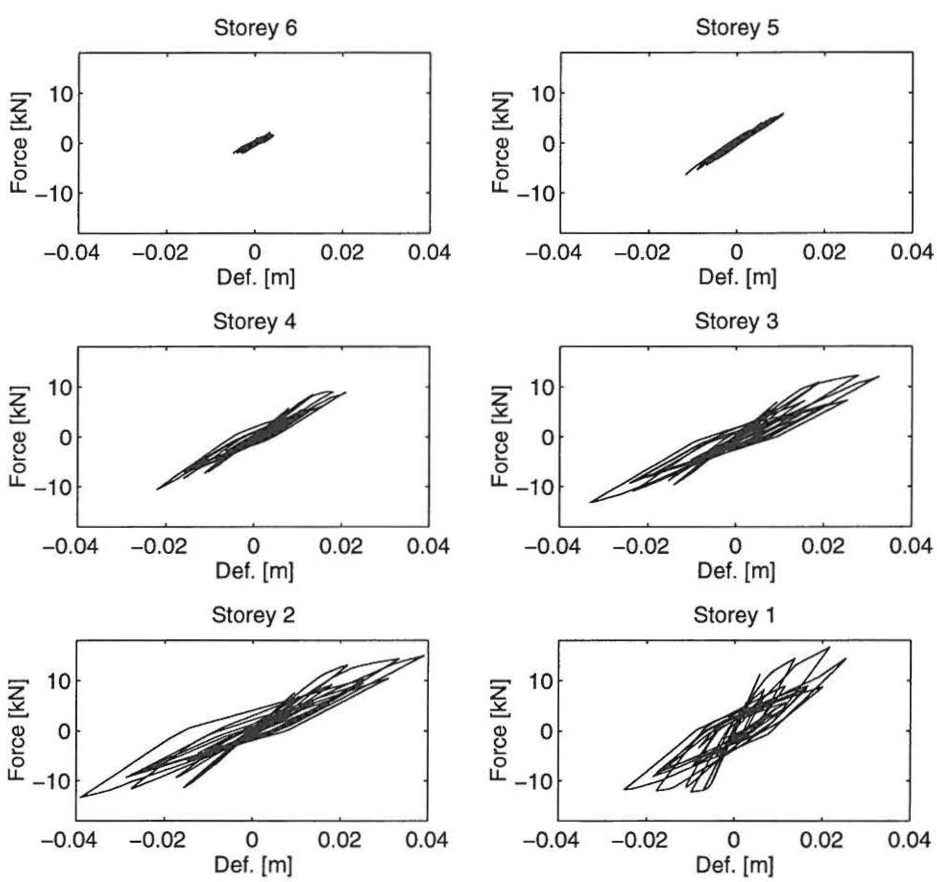

Figure 10: Estimated force-deformation curves.

From table 1 it is seen that all methods generally show very small damage in the two upper storeys which is in very good agreement with the calculated SARCOF damage indicators for these storeys. The SARCOF damage indicators for the four lower storeys show damage in all storeys, with decreasing damage level from storey one which is the most damaged storey, down to storey four which is the less damaged. The normalized dissipated energy (NDE), the flexural damage ratio (FDR), the damage index by Stephens (SEI) and the LSDI show the same tendency, whereas the interstorey drift ID and the Park and Ang index P& A show maximum damage in storey two. It should here be noted that the LSDI has been calculated from only one response measurement since this damage index is calculated from frequency information only.

4 Conclusions

From the studies performed in this paper it is seen that a good correlation is found between the damage index δ_{SARCOF} and the damage index DSI defined by Stephens, the flexural damage ratio FDR and the local softening damage index $LSDI$. The interstorey drifts ID, the normalized cumulative deformation NCD, Park and Ang's damage index P& A and the normalized dissipated energy NDE show somewhat poorer correlation to the damage index δ_{SARCOF}.

For the case considered, it can be concluded that the calculated DIs indicate that the two lower storeys are severely damaged, the third and fourth storey are a little less damaged and the fifth and sixth storey are practically undamaged.

5 Acknowledgement

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